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THE NEW HEC-1 FLOOD HYDROGRAPH PACKAGE

by

ARLEN D. FELDMAN

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THE HYDROLOGIC ENGINEERING CENTER
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> This paper describes the new version of the HEC-1 Flood Hydrograph Package-a comprehensive simulation model computer program. The many hydrologic and hydraulic simulation capabilities of the model are described. Special emphasis is given to analysis of dam safety and dam failure flood damage studies, and urban hydrology. The general applicability and usage of the model are described.

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ABSTRACT

HEC-1 is a mathematical watershed model containing several methods with which to simulate surface runoff and river/reservoir flow in river basins. The hydrologic model together with flood damage computations (also included in the model) provide a basis for evaluation of flood control projects. HEC-1 was developed by the Hydrologic Engineering Center (HEC), U.S. Army Corps of Engineers, in the late 1960's; a new version of the model, with greatly expanded capabilities, was released in 1980 and is described in this paper. The capabilities of the new HEC-1 Flood Hydrograph Package include: simulation of rainfall and/or snowmelt runoff from subbasins and flow through a stream network, simulation of flows in urban areas, hydrologic calculations for dam safety and dam failure studies, and economic calculations for planning flood control systems.

HEC-1 simulates a stream network using four components: 1) runoff from a subbasin, 2) hydrograph routing, 3) combining of hydrographs, and 4) flow diversion. Most complex, branching stream networks can be simulated with the model. The various options for watershed runoff calculation are described including: precipitation, interception/infiltration, precipitation excess-to-runoff transformation, river routing, and flow through reservoirs. Diversions and multistage pumping plants capabilities are also described. Flow in urban areas can be simulated using kinematic wave routing of rainfall excess along a path which includes overland flow elements, collector channels, and a main channel to a subbasin outlet. A special routing routine is described for simulating flow through a dam and spillway, over the top of dam, or through a dam breach. This can be used in conjunction with other stream network modeling capabilities to determine potential hazards from dam overtopping or failure. This capability has been frequently used in the U.S. National Non-Federal Dam Safety Inspection Program.

In addition to its hydrologic capabilities, HEC-1's application to economic evaluation of flood hazards and flood control systems is presented. Expected annual flood damage is computed using the watershed model results together with flood-frequency and flood-damage data. Flood damage may be calculated for any locations in the river basin and for existing and alternative flood control projects. When damage estimates are combined with cost data for the projects and a systematic search procedure, the model can provide an estimate of the optimal size of the flood control projects based on maximum net benefits. This enables a planner to select the most desirable flood control scenario.

INTRODUCTION

History of HEC-1

The HEC-1, Flood Hydrograph package, computer program was originally developed in 1967 by Leo R. Beard and other members of the Hydrologic Engineering Center staff to simulate flood hydrology in complex river basins. The first package version represented a combination of several smaller programs which had previously been operated independently to simulate various aspects of the rainfall/snowmelt process. In 1973, the program underwent a major revision. The computational methods used by the program remained basically unchanged; however, the input and output formats were almost completely restructured. These changes were made in order to simplify input requirements and to make the program output more meaningful and readable.

The present program (HEC, 1981a) again represents a major revision of the 1973 version of the program. The program input and output formats have been completely revised and the computational capabilities of the dam-break (HEC-1DB), project optimization (HEC-1GS) and kinematic wave (HEC-1KW) programs have been combined in the one program. The new program gives the powerful analysis features available in all the previous programs, together with some additional capabilities, in a single easy-to-use package.

Purpose of HEC-1

The HEC-1 model is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin with interconnected hydrologic and hydraulic components. It is primarily applicable to flood simulation. English or metric units may be used. Each component models an aspect of the precipitation-runoff process within a portion of the basin, commonly referred to as a subbasin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters which specify the particular characteristics of the component and mathematical relations which describe the physical processes. The result of the modeling process is the computation of streamflow hydrographs at desired locations in the river basin.

The flood hydrograph information provided by HEC-1 has been extensively used in flood plain information studies and flood control project evaluations. The interconnection of HEC-1's hydrologic outputs with water surface profile and reservoir operation models and flood damage analyses was described by Feldman (1981). The other water resources system simulation models of the Hydrologic Engineering Center are also described in that publication.

COMPONENTS OF THE MODEL

The stream network model is the basic foundation capability of the HEC-1 program. All other program computation options build on this option's capability to calculate flood hydrographs at desired locations in a river basin. This section discusses: the conceptual aspects of using the HEC-1 program to formulate a stream network model from basic river basin data; model formulation as a step-by-step process; and the

functions of each component in representing individual characteristics of the river basin.

Stream Network Model Development

A river basin is subdivided into an interconnected system of stream network components using topographic maps and other geographic information. A basin schematic diagram (e.g., Fig. 1) of these components is developed by the following steps:

(1) The study area watershed boundary is delineated first. In a natural or open area this can be done from a topographic map. However, supplementary information, such as municipal drainage maps, may be necessary to obtain an accurate depiction of an urban basin's extent.

(2) Segmentation of the basin into a number of subbasins determines the number and types of stream network components to be used in the model. Two factors impact on the basin segmentation: the study purpose and the hydrometeorological variability throughout the basin. First, the study purpose defines the areas of interest in the basin, and hence, the points where subbasin boundaries should occur. Second, the variability of the hydrometeorological processes and basin characteristics impact greatly on the number and location of subbasins. Each subbasin is intended to represent an area of the watershed which, on the average, has the same hydraulic/hydrologic properties. Further, the assumption of uniform precipitation and infiltration over a subbasin becomes less accurate as the subbasin becomes larger. Consequently, if the subbasins are chosen appropriately, the average parameters used in the components will more accurately model the subbasins. The number of subbasins used also has a direct effect on the cost of the model. Consequently, it pays to be as economical as possible with the number of subbasins.

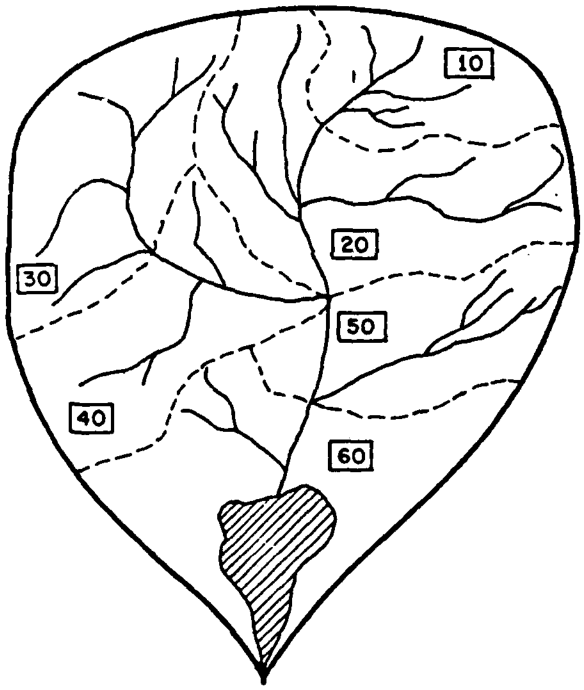
(3) Each subbasin is to be represented by a combination of model components. Subbasin runoff, river routing, reservoir and diversion and pump components are available to the user.

(4) The subbasins and their components are linked together to represent the connectivity of the river basin. HEC-1 has available a number of methods for combining or linking together outflow from different components. This step finalizes the basin schematic.

Land Surface Runoff Component

The subbasin land surface runoff component, such as subbasins 10, 20, 30, etc. in Fig. 1, is used to represent the movement of water over the land surface and in stream channels. Inputs to this component can be a precipitation hyetograph and a soil water infiltration rate function. Note that the rainfall and infiltration are assumed to be uniform over the subbasin. The infiltration losses are subtracted from the rainfall and the resulting rainfall excesses are then routed by the unit hydrograph or kinematic wave techniques to the outlet of the subbasin producing a runoff hydrograph. The unit hydrograph technique produces a runoff hydrograph at a discrete point, usually the most downstream point in the subbasin. If this location for the runoff

computation is not appropriate, it may be necessary to further subdivide the subbasin or use the kinematic wave method to distribute the local inflow. The kinematic wave rainfall excess-to-runoff transformation allows for the uniform distribution of the land surface runoff along the length of the main channel. This uniform distribution of local inflow (subbasin runoff) is particularly important in areas where many lateral channels contribute flow along the length of the main channel.



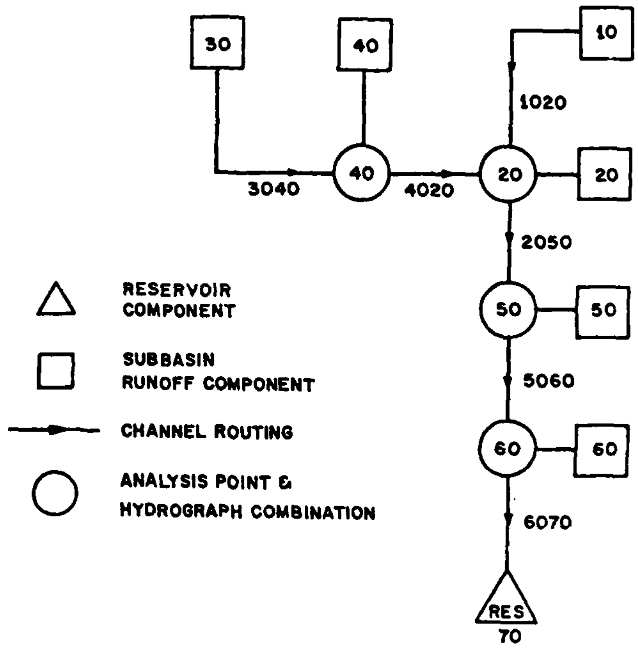


Figure 1 HEC-1 DEPICTION OF A RIVER BASIN

River Routing Component

A river routing component, element 1020, Fig. 1, is used to simulate the flow of water in a river channel. The input to the component is an upstream hydrograph resulting from subbasin runoff, river routings or combinations of both. If the kinematic wave method is used, the local subbasin distributed runoff is also input to the main channel and combined with the upstream hydrograph as it is routed to the end of the reach. The hydrograph is routed to a downstream point based on the characteristics of the channel.

Combined Use of River Routing and Subbasin Runoff Components

Consider the use of subbasin runoff components 10 and 20 and river routing reach 1020 in Fig. 1. The runoff from component 10 is calculated and routed to control point 20 via routing reach 1020. Runoff from subbasin 20 is then calculated and combined with the outflow hydrograph from reach 1020 at control point 20. Note that this method of adding flows approximates the addition of lateral inflow to reach 1020. The runoff from subbasin 20 could be calculated directly at control point 20 in a unit hydrograph subbasin runoff calculation, or it could have been uniformly distributed along reach 1020 in a kinematic wave subbasin runoff calculation. A suitable combination of the subbasin runoff component and river routing components can be used to represent the intricacies of any rainfall-runoff and stream routing problem. The connectivity of the stream network components is implied by the order in which the data components are arranged. Simulation must always begin at the uppermost subbasin in a branch of the stream network. The simulation (succeeding data components) proceeds downstream until a confluence is reached. Before simulating below the confluence, all flows above that confluence must be computed and routed to that confluence. The flows are combined at the confluence and the combined flows are routed downstream. In Fig. 1, all flows tributary to control point 20 must be combined before routing through reach 2050.

Reservoir Component

The reservoir component can be used to represent the storage-outflow characteristics of a reservoir, lake, detention pond, highway culvert, etc. The reservoir component application is similar to that of the river routing component. Upstream inflows are routed through a reservoir based on the specified storage outflow characteristics as is the case in some river routing options. Consequently, the same flood routing methods can be applied for either component.

Diversion Component

The diversion component is used to represent channel diversions, stream bifurcations, or any transfer of flow from one point of a river basin to another point in or out of the basin. The diversion component receives an upstream inflow and divides the flow according to a user-prescribed rating curve.

PRECIPITATION-RUNOFF SIMULATION

The HEC-1 model components are used to simulate the precipitation-runoff process as it occurs in an actual river basin. The model components function based on simple mathematical relationships which are intended to represent individual meteorologic, hydrologic and hydraulic processes which comprise the precipitation-runoff process. These processes are separated into precipitation, interception/infiltration, transformation of precipitation excess to subbasin outflow, addition of baseflow and flood hydrograph routing.

Precipitation

A precipitation hyetograph is used as the input to all runoff calculations. The specified precipitation is assumed to be a subbasin average (i.e., uniformly distributed over the subbasin). Any of the model options used to specify precipitation will eventually result in a hyetograph. The hyetograph represents subbasin average precipitation depths over a computation interval. Precipitation data for an observed storm event can be supplied to the program by either of two methods: subbasin-average, or gages and weightings.

There are three methods for generating synthetic storm distributions: standard project, probable maximum, and specific frequency storms. The Standard Project Storm (Corps of Engineers, 1952) has a duration of 96 hours. The percentages of the index precipitation falling during each 24-hour period of the storm are automatically calculated by HEC-1 according to the Corps criteria. Probable Maximum Precipitation (National Weather Service, 1956) may be simulated for a minimum of 24 hours and up to 96 hours. The day with the largest amount of precipitation is preceded by the second largest and followed by the third largest. The fourth largest precipitation day precedes the second largest. The distribution of 6-hour precipitation during each day is according to standard criteria of the Weather Service or the Corps.

A synthetic storm of any duration from 5 minutes to 10 days can be generated based on given depth-duration data (National Weather Service, 1961). Depth for 10-minute and 30-minute durations are interpolated from 5-, 15- and 60-minute depths using equations from HYDRO-35 (National Weather Service, 1977). Cumulative precipitation for each time interval is computed by log-log interpolation of depths from the depth-duration data. Incremental precipitation is then computed and rearranged so the second largest value precedes the largest value, the third largest value follows the largest value, the fourth largest precedes the second largest, etc.

Snowfall and Snowmelt

Where snowfall and snowmelt are considered, there is provision for separate computation in up to ten elevation zones within a subbasin. These zones may be of any equal increments of elevation with a corresponding air temperature lapse rate per zone. The input temperature data are those corresponding to the bottom of the lowest elevation zone. Temperatures are reduced by the lapse rate in degrees per increment of elevation zone. The base temperature at which melt will occur, must be specified because variations from $0^{o}C$ ($32^{o}F$) might be warranted considering both spatial and temporal fluctuations of temperature within the zone. Precipitation is assumed to fall as snow if the zone temperature is less than the base temperature plus 2 degrees. Melt occurs when the zone temperature is equal to or greater than the base temperature. Snowmelt is subtracted from and snowfall is added to the snowpack in each zone. Snowmelt may be computed by the degree-day or energy-budget methods. The basic equations for snowmelt computations are from EM 1110-1-1406 (Corps, 1960b). The energy-budget equations have been simplified for use in this program.

Interception/Infiltration

Land surface interception, depression storage and infiltration are referred to in the HEC-1 model as precipitation loss rate computations. Interception and depression storage are intended to represent the surface storage of water by trees or grass, local depressions in the ground surface, in cracks and crevices in parking lots or roofs, or in a surface area where water is not free to move as overland flow. Infiltration represents the movement of water to areas beneath the land surface.

Two important factors should be noted about the precipitation loss computation in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery (the Holtan loss rate option is an exception in that soil moisture recovery occurs by percolation out of the soil moisture storage). This fact dictates that the HEC-1 program is a single-event-oriented model.

The precipitation loss is considered to be a subbasin average (uniformly distributed over an entire subbasin). For the kinematic wave runoff transformation separate precipitation losses can be specified for two types of overland flow planes. The losses are assumed to be uniformly distributed over each overland flow plane. In some instances, there are negligible precipitation losses occurring for a portion of a subbasin. This would be true for an area containing a lake, reservoir or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

There are four methods that can be used to calculate the precipitation loss. Using any one of the methods, an average precipitation loss is determined for a computation interval and subtracted from the rainfall/snowmelt hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a subbasin.

An initial loss (units of depth) and a constant loss rate (depth/hour) is the first option. All rainfall is lost until the volume of initial loss is satisfied. After the initial loss is satisfied, rainfall is lost at the constant rate. The second method is the HEC Exponential Loss Rate Method. This is an empirical method which relates loss rate to rainfall intensity and accumulated losses. Accumulated losses are representative of the soil moisture storage. Estimates of the parameters of the exponential loss function can be obtain- ed by employing the HEC-1 parameter optimization option described in a later section. A similar loss rate function is used for snowmelt.

The Soil Conservation Service (SCS), U.S. Department of Agriculture, has instituted a loss rate technique which relates the drainage characteristics of soil groups to a curve number, CN (SCS, 1965 and 1975). The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions. Precipitation loss is calculated based on supplied values of CN and an initial surface moisture storage capacity in units of depth. Since the SCS method gives total excess for a storm, the incremental excess (the difference between rainfall and loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

The fourth loss rate option is a method developed by Holtan et al. (1975). It computes loss rate based on the infiltration capacity given by the formula:

$$f = f_c + G*a*s^b \qquad (1)$$

where f is the infiltration capacity in inches per hour, G is a growth index representing the relative maturity of the ground cover, a is the infiltration capacity in inches per hour per (inch of available storage)^b, s is the equivalent depth in inches of pore space in the surface layer of the soil which is available for storage of infiltrated water, f_c is the constant rate of percolation of water through the soil profile below the surface layer, and b is an empirical exponent, typically taken equal to 1.4.

Precipitation Excess-to-Runoff Transformation

HEC-1 provides two methods for transforming rainfall/snowmelt excesses into runoff: unit hydrograph and kinematic wave. The unit hydrograph technique has been discussed extensively in the literature (Linsley et al., 1975, and Viessman et al., 1977). This technique is used in the subbasin runoff component to transform rainfall/snowmelt excess to subbasin outflow. A unit hydrograph can be directly input to the program or a synthetic unit hydrograph can be computed from user supplied parameters. The parameters for the synthetic unit hydrograph can be determined from gage data by employing the parameter optimization option described in a later section. Otherwise, these parameters can be determined from regional studies or from guidelines given in references for each synthetic technique. There are three synthetic unit hydrograph methods available in the model. The synthetic techniques compute the unit graph for whatever computational time interval is being used in the simulation.

The Clark method (1945) requires three parameters to calculate a unit hydrograph: the time of concentration for the basin, a storage coefficient, and a time-area curve. The time-area curve defines the cumulative area of the watershed contributing runoff to the subbasin outlet as a proportion of the time of concentration. In the case that at time-area curve is not supplied, the program utilizes a synthetic elliptical time-area curve. The Snyder method (1938) determines the unit graph peak discharge, time to peak, and widths of the unit graph at 50 and 75% of the peak discharge. The method does not produce the complete unit graph required by HEC-1. Thus, HEC-1 uses the Clark method to produce a Snyder unit graph. The Soil Conservation Service dimensionless unit hydrograph method (1965) uses a single parameter, which is equal to the lag (hours) between the center of mass of rainfall excess and the peak of the unit hydrograph. Peak flow is computed using subbasin area and time to peak. The unit hydrograph ordinates are computed from a dimensionless graph using the peak flow and time to peak.

The kinematic wave subbasin runoff method in HEC-1 (HEC, 1979b) is composed of three elements: overland flow planes, collector channels, and a main channel, Fig. 2. Through these elements, the kinematic wave technique transforms rainfall excess into subbasin outflow. This simulation may be done on a detailed street-by-street basis in an urban area or set up to simulate representative drainage systems within a subbasin. If a representative system is used, the program automatically computes the total subbasin runoff as a function of the area of the representative system and the total area of the subbasin.

In the kinematic wave interpretation of the equations of fluid motion the momentum equation is reduced to a stage-discharge relation. The wave characteristics of a flood are then described solely by the continuity equation. HEC-1 solves the kinematic wave equations using a finite difference algorithm based on the same method developed for the MITCAT simulation model (Harley, 1975). Detailed development of the specific finite difference equations, the coding procedures and boundary requirements can be found in the following references: Harley, 1975; and Hydrologic Engineering Center, 1979b.

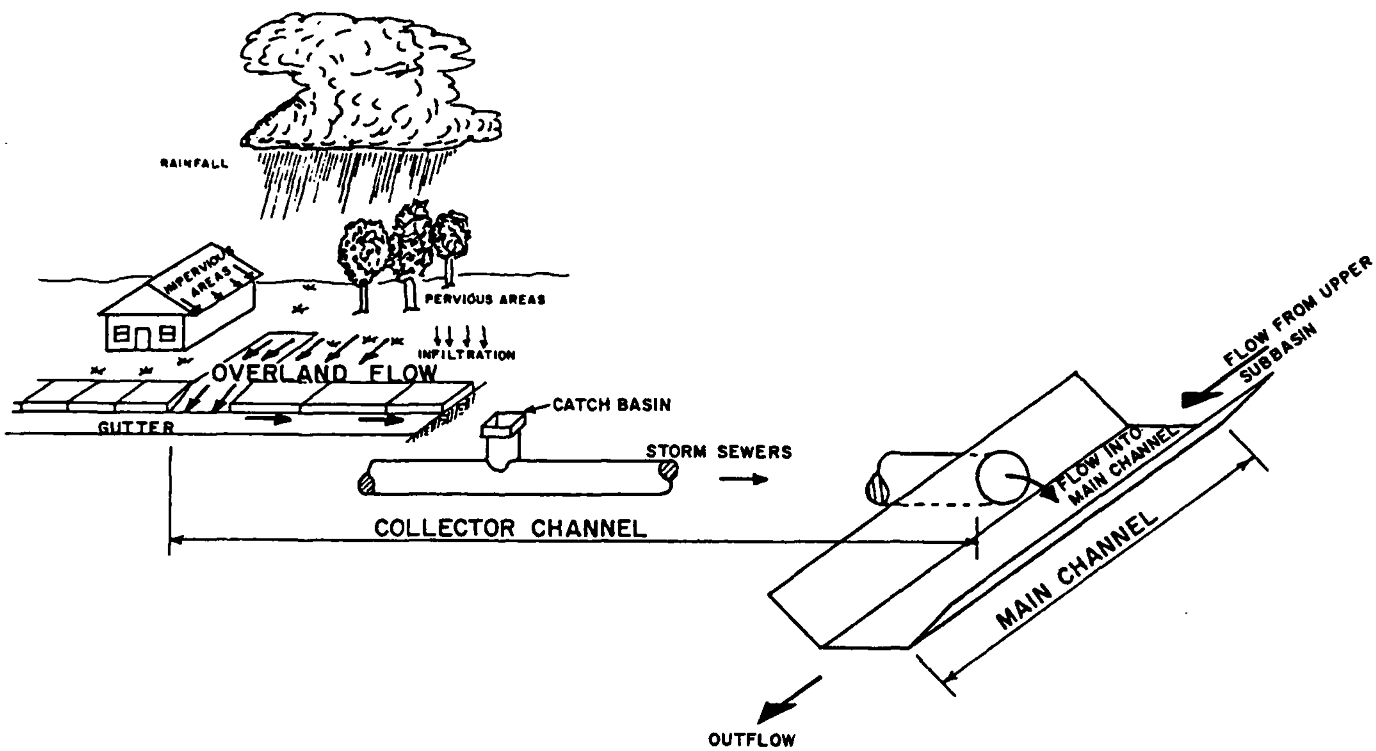


Figure 2 DEPICTION OF KINEMATIC WAVE RUNOFF (figure 1.5 : HEC, 1979b)

Base Flow

Two distinguishable contributions to a flood hydrograph are direct runoff (described earlier) and base flow which results from releases of water from surface and subsurface storage. The HEC-1 model provides means to include the effects of base flow on the streamflow hydrograph as a function of three input parameters: starting flow, a recession threshold, and a recession rate as shown in Fig. 3. Both the initial and base flow recession occur at an exponential decay rate, which is assumed to be a characteristic of the basin. The rising limb of the streamflow hydrograph is adjusted for base flow by adding the recessed starting flow and computed direct runoff flows. The falling limb is determined in the same manner until the computed flow is determined to be less than the threshold. From this time on, the streamflow hydrograph is computed using the recession equation unless the computed flow rises above the base flow recession. This is the case of a double-peaked streamflow hydrograph where the rising limb of the second hydrograph is computed as before, using the recessed starting flow and the computed direct runoff.

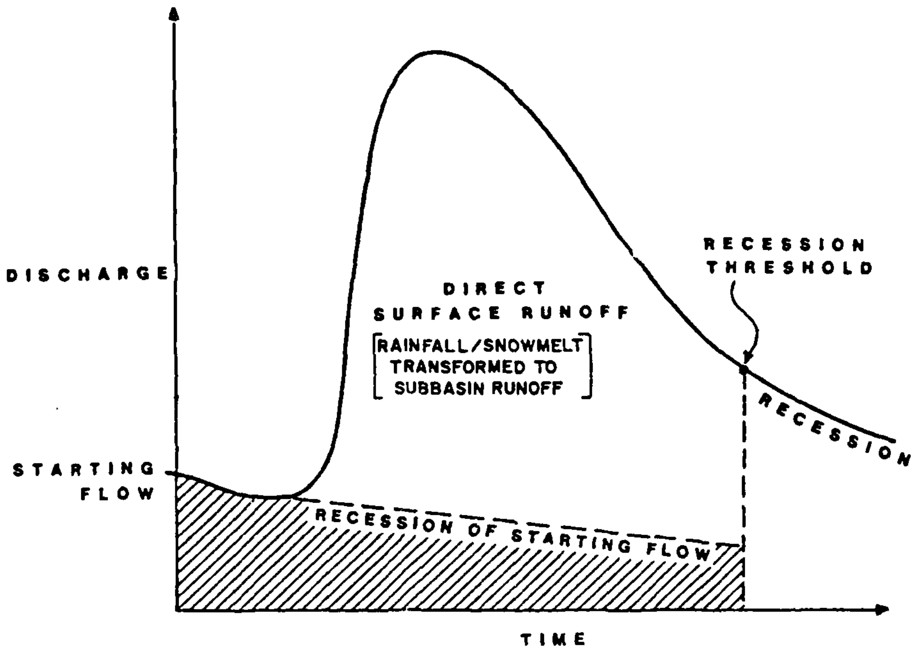


Figure 3 HEC-1 SUBBASIN RUNOFF SIMULATION

Flood Routing

Flood routing is used to simulate the outflows from river reaches and reservoirs. Most of the flood routing methods available in HEC-1 are based on the continuity equation and some relationship between flow and storage or stage. These methods are Muskingum, kinematic wave, modified Puls, working R and D, and level-pool reservoir routing. In all of these methods, routing proceeds on an independent reach basis from upstream to downstream; backwater effects are not considered. These methods cannot simulate discontinuities in the water surface such as jumps or bores. These methods should, however, give good results for routing floods through channels on moderate to steep slopes and through reservoirs. There are also two routing methods in HEC-1 (Tatum and Straddle-Stagger) which are based on lagging averaged hydrograph ordinates. These methods are not physically based, but have been used on several rivers with good results. Channel infiltration losses may be simulted. Hydrographs are adjusted for losses after routing for all methods except modified Puls; for modified Puls, losses are computed before routing.

The Muskingum method (Chow, 1964) computes outflow from a reach as a function of the current period inflows and the previous period inflow and outflow. The routing procedure may be repeated for several subreaches. The total travel time through the reach and the magnitude of the wedge storage coefficient are checked by the program for physical and computational constraints.

Storage routing methods in HEC-1 are those methods which require data about the storage characteristics of a routing reach or reservoir. These methods are: modified Puls, working R and D, and level-pool reservoir routing. These methods also require outflow data which is related to storage. There are three methods for determining routing reach storage in HEC-1: (1) direct input, (2) surface area and elevation for reservoirs (conic method), and (3) channel cross-section and reach length (normal-depth channel flow). Outflow characteristics can be computed from: direct input, normal-depth channel flow, weir equation (spillway), critical depth (trapezoidal spillway), or ogee spillway data. Whenever storage and outflow data are computed from methods other than direct input, elevation (stage) data must be supplied so the relation between storage and outflow can be determined. If the storage routing procedure used is the modified Puls (given storage versus outflow or computed by normal-depth channel flow), the working R&D, or the trapezoidal spillway critical depth method, a storage versus outflow relationship is first computed from the input data and then used in all time-interval computations. If the level-pool reservoir routing with low-level orifice and weir spillway outflows is used, storage and outflow are computed from the current reservoir water surface elevation in each time interval.

Storage and outflow data for use in storage routing may be computed from channel characteristics. The program uses an 8-point cross section which is representative of the routing reach. Outflows are computed for normal depth using Manning's equation. Storage is cross-sectional area times reach length. Storage and outflow values are computed for 20 evenly-spaced stages beginning at the lowest point on the cross section to a specified maximum stage. The cross section is extended vertically at each end to the maximum stage.

The modified Puls routing method (Chow, 1964) is a variation of the storage routing method described by Henderson (1966). A storage indication function is computed from given storage and outflow data. The outflow at the end of the time interval is interpolated from a table of storage indication versus outflow. Storage is then computed from a continuity relationship. When stage data are given, stages are interpolated for computed storages. Initial conditions can be specified in terms of storage, outflow, or stage. The corresponding value of storage or outflow are computed from the given initial value. The working R and D method (Corps of Engineers, 1960a) is a variation of modified Puls method which accounts for wedge storage as in the Muskingum method.

Level-pool reservoir routing assumes a level water surface in a reservoir. It is used in conjunction with the pump option described subsequently and with the dam-break calculation described in a later section. Using the principle of conservation of mass, the change in reservoir storage for a given time period is equal to average inflow minus average outflow. An iterative procedure is used to determine end-of-period storage and outflow. Pumps may be included as a part of level-pool reservoir routing. The program checks the reservoir stage at the beginning of each time period. If the stage exceeds the "pump-on" elevation the pump is turned on and the pump output is included as an additional outflow term in the routing equation. When the reservoir stage drops below a "pump-off" elevation, the pump is turned off. Several pumps with different on and off elevations may be used. Each pump discharges at a constant rate. Pumped flow is lost from the system and is not available for any further calculations.

Reservoir outflow for storage routing may be computed from a description of the outlet works (low-level outlet and spillway). There are two subroutines in HEC-1 which compute outflow rating curves. The first uses simple orifice and weir-flow equations while the second computes outflow from specific energy or design graphs and corrects for tailwater submergence. An outflow rating curve is computed for 20 elevations which span the range of elevations given for storage data. Storages are computed for these outflows and this storage versus outflow relation is used for modified Puls or working R and D routing. For level-pool reservoir routing outflows are computed for the orifice and weir equations for each routing interval. Trapezoidal and ogee spillways (Corps of Engineers, 1963) may also be simulated using appropriate pier and abutment losses.

Kinematic wave channel routing can be utilized independently of the other elements of the subbasin runoff. In this case, upstream inflow is routed through a reach (independent of lateral inflows) using the previously described kinematic wave methods.

PARAMETER OPTIMIZATION

Calibration and verification are essential parts of the modeling process. Rough estimates for the parameters in the HEC-1 model can be obtained from the literature, however, the model should be calibrated to observed flood data whenever possible. HEC-1 provides a powerful optimization technique for the estimation of some of the parameters when gaged precipitation and runoff data are available. By using this technique and regionalizing the results, rainfall-runoff parameters for ungaged areas can also be estimated (HEC, 1981b). A summary of the HEC's experience with automatic calibration of rainfall-runoff models is given by Ford et al. (1980).

The parameter optimization option has the capability to automatically determine a set of unit hydrograph and loss rate parameters that "best" reconstitute an observed runoff hydrograph for a subbasin. The data which must be provided to the model are: basin average precipitation; basin area; starting flow and base flow parameters; and the outflow hydrograph. Unit hydrograph and loss rate parameters can be determined individually or in combination. Parameters that are not to be determined from the optimization process must be estimated and provided to the model. Initial estimates of the parameters to be determined can be input by the user or chosen by the program's optimization procedure.

The runoff parameters that can be determined in the optimization are the unit hydrograph parameters of the Snyder, Clark and SCS methods and loss rate parameters of the exponential, Holtan, SCS, and initial/constant methods. The melt rate and threshold melt temperature can also be optimized for snow hydrology studies.

The "best" reconstitution is considered to be that which minimizes the weighted squared difference between the observed hydrograph and the computed hydrograph. Presumably, this difference will be a minimum for the optimal parameter estimates. The sum of the weighted squared differences STDER objective function is defined as follows:

$$STDER = \sum_{i=1}^{n} (QOBS_i - QCOMP_i)^2 * WT_i/n \quad \ldots\ldots\ldots\ldots (2)$$

where $QCOMP_i$ is the runoff hydrograph ordinate for time period i computed by HEC-1, $QOBS_i$ is the observed runoff hydrograph ordinate i, n is the total number of hydrograph ordinates, and WT_i is the weight for the hydrograph ordinate i computed from the following equation:

$$WT_i = (QOBS_i + QAVE) / (2*QAVE) \quad \ldots\ldots\ldots\ldots (3)$$

where QAVE is the average computed discharge. This weighting function emphasizes accurate reproduction of peak flows rather than low flows by biasing the objective functions. Any errors for computed discharges that exceed the average discharge will be weighted more heavily, and hence the optimization scheme should focus on reduction of these errors.

The minimum of the objective function is found by employing the univariate search technique (Ford et al., 1980). The univariate search method computes values of the objective function for various values of the optimization parameters. The values of the parameters are systematically altered until STDER is minimized. The range of feasible values of the parameters is bounded because of physical limitations on the values that the various unit hydrograph, loss rate, and snowmelt parameters may have, and also because of numerical limitations imposed

by the mathematical functions. The optimization procedure does not guarantee that a "global" optimum (or a global minimum of the objective function) will be found for the runoff parameters; a local minimum of the objective function might be found by the procedure. To help assess the results of the optimization, HEC-1 provides graphical and statistical comparisons of the observed and computed hydrographs. From this, the user can then judge the accuracy of the optimization results.

HEC-1 may also be used to automatically derive routing criteria for certain hydrologic routing techniques. Criteria can be derived for the Tatum, straddle-stagger and Muskingum routing methods. Observed hydrographs are reconstituted to minimize the squared sum of the deviations between the observed hydrograph and the reconstituted hydrograph. The procedure used is essentially the same as for the unit hydrograph and loss rate parameter optimization.

MULTIPLAN-MULTIFLOOD ANALYSIS

The multiplan-multiflood simulation option allows a user to investigate a series of floods for a number of different characterizations (plans) of the watershed in a single computer run. The advantage of this option is that multiple storms and flood control projects simulations can be performed in a single computer run and the results compared with a minimum of effort by the user.

The multiflood simulation allows the user to analyze several different floods in the same computer run. The floods are specified as fractions of a base event (e.g., 0.5, 1.0, 1.5, etc.) which may be of either precipitation or runoff. In the case of rainfall, each ordinate of the input base-event hyetograph would be multiplied by a ratio and a stream network rainfall-runoff simulation carried out for each ratio. This is done for every ratio of the base event. In the case of runoff ratios, the ratios are applied to the computed or direct-input hydrograph and no rainfall-runoff calculations are made for individual ratios.

The multiplan option allows a user to conveniently modify a basin model to reflect desired flood control projects and changes in the basins's runoff response characteristics. This is useful when, for example, a comparison of flood control options or the effects of urbanization are being analyzed. The user designates PLAN 1 as the existing river basin model, and then modifies the existing plan data to reflect basin changes (such as reservoirs, channel improvements, or changes in land use) in PLANS 2, 3, etc. If the basin's rainfall-runoff response characteristics are modified in one of the plans, precipitation ratios and not runoff ratios must be used. Otherwise, ratios of hydrographs should be used. The program performs a stream network analysis, or multiflood analysis, for each plan. The results of the analysis provide flood hydrograph data for each plan and each ratio of the base event. The summary of the results at the end of the program output provides the user with a convenient method for comparing the differences between plans (alternative flood control systems).

DAM SAFETY/FAILURE ANALYSIS

The dam failure analysis capability was added to the HEC-1 model to assist in studies required for the United States National Non-Federal Dam Safety Program. This option uses simplified hydraulic techniques to estimate the potential for and consequences of dam overtopping or structural failures on downstream areas.

A dam failure analysis utilizes the network modelling techniques with some added capabilities for reservoir routing. These additional reservoir routing capabilities calculate flow through low-level outlets, spillway, over the top of the dam, and through a breach. The dam failure simulation differs from the previously described reservoir routing in that the stage-outflow relation is computed by determining the flow over top of the dam (dam overtopping) and/or through the dam breach (dam break) as well as through other reservoir outlet works, Fig. 4. The stage-outflow characteristics are then combined with the level-pool storage routing to simulate a dam failure.

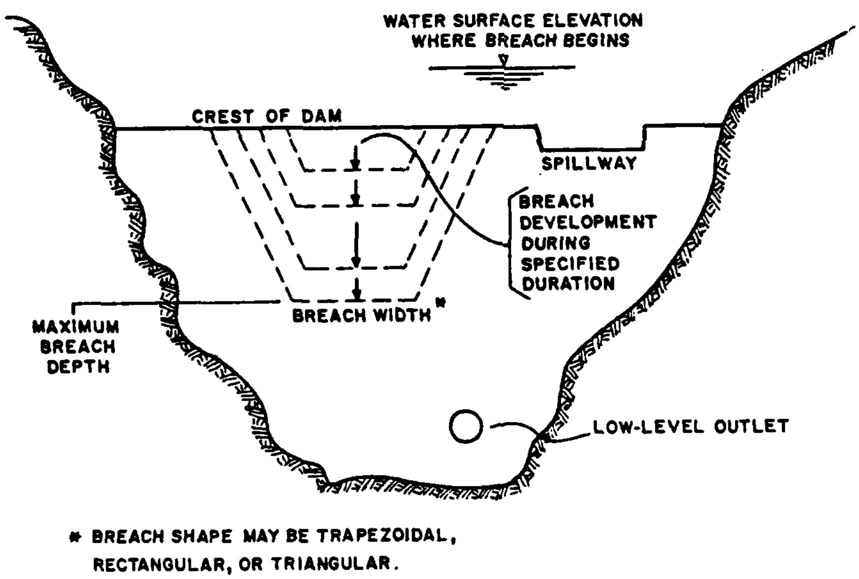


Figure 4 COMPONENTS OF NORMAL AND BREACH FLOW THROUGH A DAM

The discharge over the top of the dam is computed by a weir flow equation. Spillway discharges continue to be computed by the spillway equation even as the water surface elevation exceeds the top of the dam elevation. The weir flow for dam overtopping is added to the spillway and low-level outlet discharges. Critical flow over a non-level dam crest is computed from crest length and elevation data. A dam crest is divided into rectangular and trapezoidal sections and the flow is computed through each section. When a dam is breached the width of the breach is subracted from the crest length beginning at the lowest portion of the dam.

Dam breaks are simulated using the methodology proposed by Fread (National Weather Service, 1979). Structural failures are modeled by assuming certain geometrical shapes for the dam breach. The variables used in the analysis, as well as the dam breach shapes available in the program, were shown in Fig. 4.

Flow through a dam breach is computed as weir flow using progressively larger weirs as the breach develops. The breach is initiated when the water surface in the reservoir reaches a specified elevation. The breach begins at the top of the dam and expands linearly to the bottom of the breach and to its full width in a specified time. The failure duration is divided into 50 computation intervals. These short intervals are used to minimize routing errors during the period of rapidly changing flows when the breach is forming. Downstream routing methods in HEC-1 use a time interval which is usually greater than the time interval used during breach development. The program output shows the short-interval failure hydrograph and the location of the regular HEC-1 time intervals. It is important to be sure that the breach hydrograph is adequately described by the HEC-1 end-of-period intervals or else the downstream routings will be erroneous.

The dam-break simulation assumes that the dam-break hydrograph will not be affected by tailwater constraints and that the reservoir pool remains level. Also, HEC-1 hydrologic routing methods are assumed appropriate for the dynamic flood wave. Under the appropriate conditions, these assumptions will be approximately true. However, care should be taken in interpreting the results of the dam-break analysis. If a more accurate analysis is needed, then an unsteady flow model, such as the National Weather Service's DAMBRK (1979), should be used.

PRECIPITATION
DEPTH-AREA RELATIONSHIP SIMULATION

One of the most difficult problems of hydrologic evaluation is that of determining the effect that a project on a remote tributary has on floods at a downstream location. A similar problem is that of deriving flood hydographs, such as the standard project floods or 100-year exceedence interval floods, at a series of locations throughout a complex river basin. Both problems could require the successive evaluation of many storm centerings upstream of each location of interest.

Since the average depth of precipitation over a tributary area for a storm generally decreases with the size of contributing area, it would ordinarily be necessary to recompute a decreasing consistent flood quantity contributed by each subarea to successive downstream points. In order to avoid the proliferation of hydrographs that would ensue, the depth area calculation of HEC-1 makes use of a number of hydrographs (termed "index hydrograph") computed from a range of precipitation depths throughout the river basin. The index hydrographs are computed from a set of precipitation depth-drainage area (index area) values, a time distribution of rainfall, and appropriate loss rate and unit hydrograph parameters. A consistent hydrograph is that which corresponds to the appropriate precipitation depth for the sub-basin's drainage area. The consistent hydrographs are determined by interpolating between the two index hydrographs bracketing the subareas drainage area. The stream system procedure of generating index hydrographs, interpolating, routing and interpolating, is repeated throughout a river basin for as many locations as are desired as described in the HEC-1 Users Manual (HEC, 1981a).

FLOOD CONTROL BENEFIT ANALYSIS

Flood control planning requires the ability to rationally assess the economic consequences of flood damage. The HEC-1 benefit analysis option provides the capability to assess flood damage and explore the economic benefits provided by alternative flood control measures. The benefit due to the implementation of a flood control plan is determined by computing the difference between damage occurring in a river basin with the flood control plan and without the plan. River basin damage is determined by summing the damage computed for particular areas or reaches of the basin.

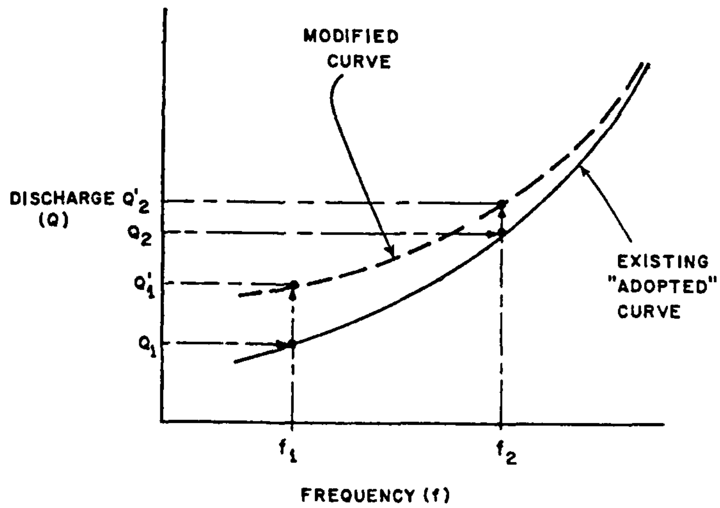
Expected annual damages (EAD) are computed as the sum of the damages weighted by a frequency of occurrence. This sum can be thought of as the average yearly damage that can be expected to occur in the reach over an extended period of time. The basic assumption of the EAD analysis is that the damage frequency curve can be obtained by combining damage versus flow (stage) and flow (stage) versus frequency relations which are characteristic of the area that the damage reach represents. The damage versus flow (stage) relation ascribes a dollar damage that occurs in an area to a level of flood flow. The flow (stage) versus exceedence frequency relation ascribes an exceedence frequency to the magnitude of flood flow. By combining this information, the damage versus frequency curve and, hence, the EAD for a reach can be determined. By comparing river basin EAD with and without flood control projects, benefits are computed as the reduction in damages.

There are two basic computations in a benefit calculation: exceedence frequency curve modification and EAD calculation. Structural flood control measures (e.g., reservoirs and channel improvements) and changes in land use affect the flow-frequency relationship. Nonstructural measures (e.g., flood proofing and warning) do not usually have much impact on the flood-frequency relationship but do modify the stage-damage relationship.

Frequency Curve Modification

The flow-exceedence frequency data provided for damage reaches refer to PLAN 1 or the base plan of the multiplan-multiflood model. Implementation of structural flood control measures or land use changes will change this exceedence frequency relation. HEC-1 computes modified frequency relationships using the following methodology. A multiflood analysis is performed for PLAN 1 to establish the frequency of the peak discharge of each ratio of the design event. The peak-flow frequency for each ratio of the design event is interpolated from the input flow-frequency data tables for a damage reach. A stage-frequency curve is established in essentially the same manner as for flows when stage-frequency data are specified for a damage reach.

A multiflood simulation is now performed for the flood control plans. The peak discharges (stages) are computed at each damage reach for each ratio of the design event. HEC-1 assumes that the frequency of each ratio remains the same as computed for the base case above; and only the peak flows associated with each ratio change for different plans. In this manner, the modified flow-frequency curve is computed for all ratios as shown in Fig. 5. That figure illustrates the potential change in a frequency curve due to urbanization. The assumption inherent in this procedure is that the event ratio-frequency relation is not affected by basin configuration.



Q_i = PEAK DISCHARGE FOR DESIGN STORM i

f_i = FREQUENCY OF Q_i FROM GIVEN CURVE

Q'_i PEAK DISCHARGE FROM DESIGN STORM i UNDER MODIFIED WATERSHED CONDITION

Figure 5 MODIFYING FREQUENCY CURVES

Expected Annual Damage (EAD) Calculation

EAD is calculated by combining the flow or stage-frequency curve and the flow- or stage-damage for each PLAN and damage reach (HEC, 1979a). The flow-frequency curve is used in conjunction with the flow-damage data to produce a damage-frequency curve as shown in Fig. 6. The area under the damage-frequency curve is the EAD for the reach. This area is computed using a three point Gaussian Quadrature formula. If more than one damage category is specified for a reach, the above steps are repeated for each land use. The EAD is summed for all the land uses to produce the EAD for the reach. The benefit accrued due to the employment of a flood control plan is equal to the difference between the PLAN 1 EAD and the flood control plan EAD. The model performs this computation for all plans in the multiplan-multiflood analysis.

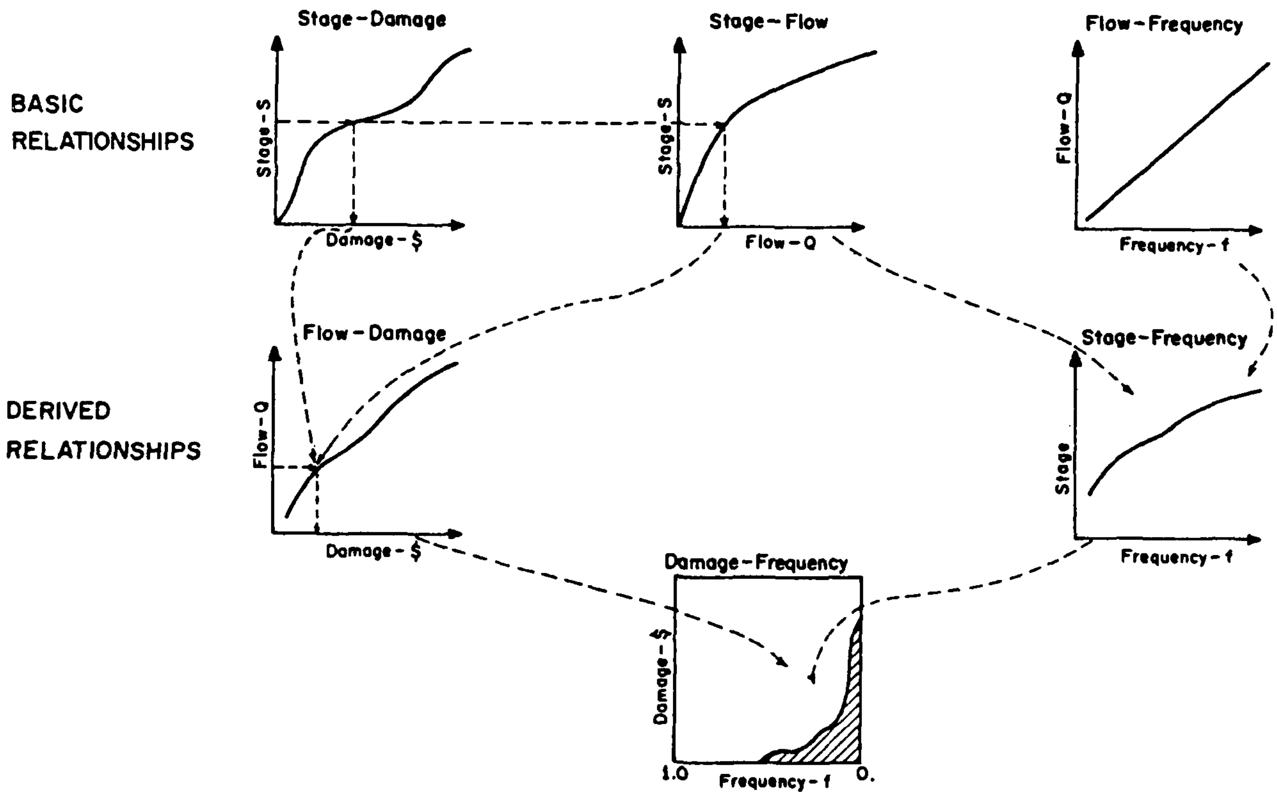


Figure 6 EXPECTED ANNUAL DAMAGE CALCULATION (figure 1 : HEC, 1979a)

FLOOD CONTROL SYSTEM OPTIMIZATION

The flood control system optimization option is used to determine optimal sizes for the flood control components in a river basin flood control plan (Davis, 1974). The optimization model is an extension of the flood damage model previously described. The optimization model utilizes a two-plan damage analysis: PLAN 1 is the base condition of the existing river basin and PLAN 2 is the flood control plan being optimized. Data on the costs of various sizes of flood control projects are required, otherwise the formulation of the optimization model is essentially the same as in the flood damage model case. The flood control components that can be optimized as part of the flood control system are as follows: reservoirs, diversions, pumping plants, and local protection projects (levees, etc.).

The storage of a reservoir may be optimized by determining the elevation of the reservoir spillway, thus defining the point where reservoir outflows are uncontrolled. The low-level outlet characteristics of the reservoir are fixed by input. Flow diversions, such as described for the stream network simulation, may have their channel capacity optimized. The diverted flow may be returned to another branch of the stream network or simply lost from the system. Pumping plants may be located virtually anywhere in a stream network and their capacity may be optimized. The pumped water is considered lost from the system and cannot be returned to another branch of the stream network. A local protection project can be used to model a channel improvement or a levee. This component can only be used in conjunction with the damage analysis of a reach because it only modifies the damage function. The local protection project analysis requires capacity and cost data together with pattern damage tables for maximum and minimum sizes of the project. Damage functions are interpolated for project sizes between these maximum and minimum design values.

The flood control component optimization model requires data as described for the flood damage model plus information about the capital and operating costs of the projects and about the objective function for the flood control scheme. The data for the various types of flood control components are essentially the same and may be separated into cost data, capacity constraints, and optimization criteria. Minimum and maximum capacity must be specified for each flood control component. An initial estimate of the size of the flood control component is also required to give the optimization procedure a starting point.

Two types of data are supplied to the program which are used to calculate the total annual cost of a flood control component. First, capacity versus capital cost tables are required to determine the capital cost for any capacity of the flood control component. A capital recovery factor is also required so that equivalent annual costs for the capital investments can be computed. Second, operation and maintenance costs are computed as a proportion of the capital cost. For pumping plants, average annual power costs for various pump capacities are required. Pump operation costs are computed in proportion to the volume pumped. Capital and operating costs for non-optimized components of the system may also be considered.

The optimization methodology can operate on maximum net benefits and/or flow targets criteria. Maximum net benefits are computed using the cost and flood damage data previously described. Desired streamflows may also be specified at any point downstream of a flood control project. These streamflow limitations, referred to as "flow targets" are specified as the flow (stage) which is desired to occur at a given frequency. For example, it may be desired to have the 5% flood at a particular location be 1,000 m^3/s. The input data for flow targets are the discharge or stage and the frequency.

The model determines an optimal flood control system by minimizing a system objective function. The system objective function is the sum of flood control system total annual cost and the expected annual damage occurring in the basin. If flow targets are specified, then the previous sum is multiplied by a penalty factor which increases the objective function proportionately to deviations from the target. Note that the minimization of the objective function leads to the maximization of the net benefits accrued due to the employment of the flood control system. Net benefits are equal to the difference between the EAD occurring in PLAN 1 and the sum of the system costs and EAD occurring in PLAN 2.

An initial system configuration is analyzed by the program based on capacities specified by the user. The model performs a stream network simulation and expected annual damage calculation for the base condition, PLAN 1, without the proposed flood control measures. The stream network and expected annual damage calculations for the initial sizes of the proposed flood control system are then calculated and the initial value of the objective function is determined. The model then uses the univariate search procedure to estimate a minimum value for the objective function. The search proceeds by using the stream network and EAD calculation to generate points on the system objective function for various flood control system capacities. These capacities are systematically altered by the procedure until an optimum is reached. As in the river basin parameter optimization, a global optimum can not be guaranteed (in fact there maybe many alternative optimal solutions). However, by inspecting the resulting net benefits provided by the system, the desirability of the optimal system can be assessed.

PROGRAM USAGE

This section describes the general organization of the input data, program output, example problems, and computer requirements.

Input Data

There are two general types of data cards for HEC-1: input control and river basin simulation data. The input control cards tell the program the format of the river basin data as well as controlling certain diagnostic output. The river basin simulation data are all identified by a unique two-character alphabetic code in card columns one and two. These codes serve two functions: they identify the data

to be read from the card; and they activate various simulation options. The first character of the code identifies the general data category and the second character identifies a specific type of data within a category. The data may be input in a free or fixed format. The stream network structure can be protrayed diagrammatically. This option causes the program to search the input data deck and determine the job step computations. A flow chart of the stream network simulation is printed.

The user may enter time series data, either hyetographs or hydrographs, at time steps other than the computation interval of the simulation. This option is convenient when entering data generated by another program or in a separate HEC-1 simulation. In many instances, certain physical characteristics are the same for a number of subbasins in the stream network model (for instance, infiltration characteristics). Further, in a multiplan analysis, much of the PLAN 1 subbasin data remains unchanged in subsequent plans. The HEC-1 program input conventions make it unnecessary to repeat much of this information in the data deck.

Program Output

A large variety and degree of detail in the printer output are available from HEC-1. The output may be categorized in terms of input data feedback, intermediate simulation results, summary results, and error messages. The degree of detail of virtually all of the program output can be controlled by the user. The input data file for each job is read and converted from free format to fixed format and a sequence number is assigned to each line. The reformatted data can be printed so the user can see the data which are going into the main part of the program.

The data used in each hydrograph computation can be printed as well as the computed hydrograph, rainfall, storage, etc. as applicable. The sources of these data are indicated by the card identification code and input line number printed on the left side of the page. Hydrographs may be printed in tabular form and/or graphed (printer plot) with the date, time, and sequence number for each ordinate. For runoff calculations, rainfall, losses, and excesses are included in the table and plot. For snowmelt calculations, separate values of loss and excess are printed for rainfall and snowmelt. For storage routings, storage and stage (if stage data are given) are printed/plotted along with discharge.

The program produces hydrologic and economic summaries of the computations throughout the river basin. The standard program hydrologic summary shows the peak flow (stage) and accumulated drainage area for every hydrograph computation in the simulation. Economic summary data show the flood damages and benefits (also costs for project optimization) for each damage reach and for the river basin. The river basin damage/benefit results may also be summarized by two locational descriptors (e.g., river name and county name) if desired. The user

can also choose time series data at selected stations to be displayed in tables at the end of the job. Hyetographs, losses, excesses, stages, storages, and hydrographs can be printed in these tables in any desired order as specified by input control.

Example Problems

The HEC-1 Users Manual (HEC, 1981a) contains several test problems which serve both as illustrative examples of various capabilities of HEC-1 and as benchmark tests to verify that the program is working correctly. The first three example problems illustrate the most basic river basin modeling capabilities. Following these, specialized capabilities of HEC-1 are added to the basic model. The last four examples are a sequence of steps necessary to perform multiflood, multiplan, flood damage, and flood control project optimization analyses.

Computer Requirements and Support

HEC-1 requires a FORTRAN IV compiler and up to 16 input/output scratch (tape, disk, etc.) files. The computer memory required on the CDC 7600 is 115,000 words. It requires approximately 7 seconds to compile on that machine. The program has been tested on several major computers and the machine dependent code removed whenever possible. The users manual and programmers supplement describe detailed program characteristics and modifications necessary to run the program on different computer systems and to reduce memory requirements. The HEC provides user support for HEC-1 and other programs (Eichert, 1978). The program and documentation may be obtained from the HEC for the cost of reproduction and handling.

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